Chapter 11
Measurements and Observations

11-1. General

Both measurements and visual observations are of importance since the overall conclusions reached from the results of a test fill program are as much qualitative as quantitative. The importance of good diary keeping and photographic records cannot be overemphasized, especially in view of the fact that design personnel who are to use the information usually cannot be present at the site at all times. Like the layout and design of test fills, the measurements and observations made are highly dependent on the primary objectives of the fill program. Advance planning and scheduling of tests are an integral part of the overall design. In this respect, flexibility is also important, since only rarely can the test program be fully laid out beforehand and carried out with no deviations. Provisions should be made for supplemental tests and for relocation, if necessary, of the test sites. Personnel who are to conduct the tests should be made familiar with the program and procedures. Personnel should also be made aware of what is expected of them as far as visual observations are concerned. It is highly desirable for a representative of the design group to be present at all times.

11-2. Densification

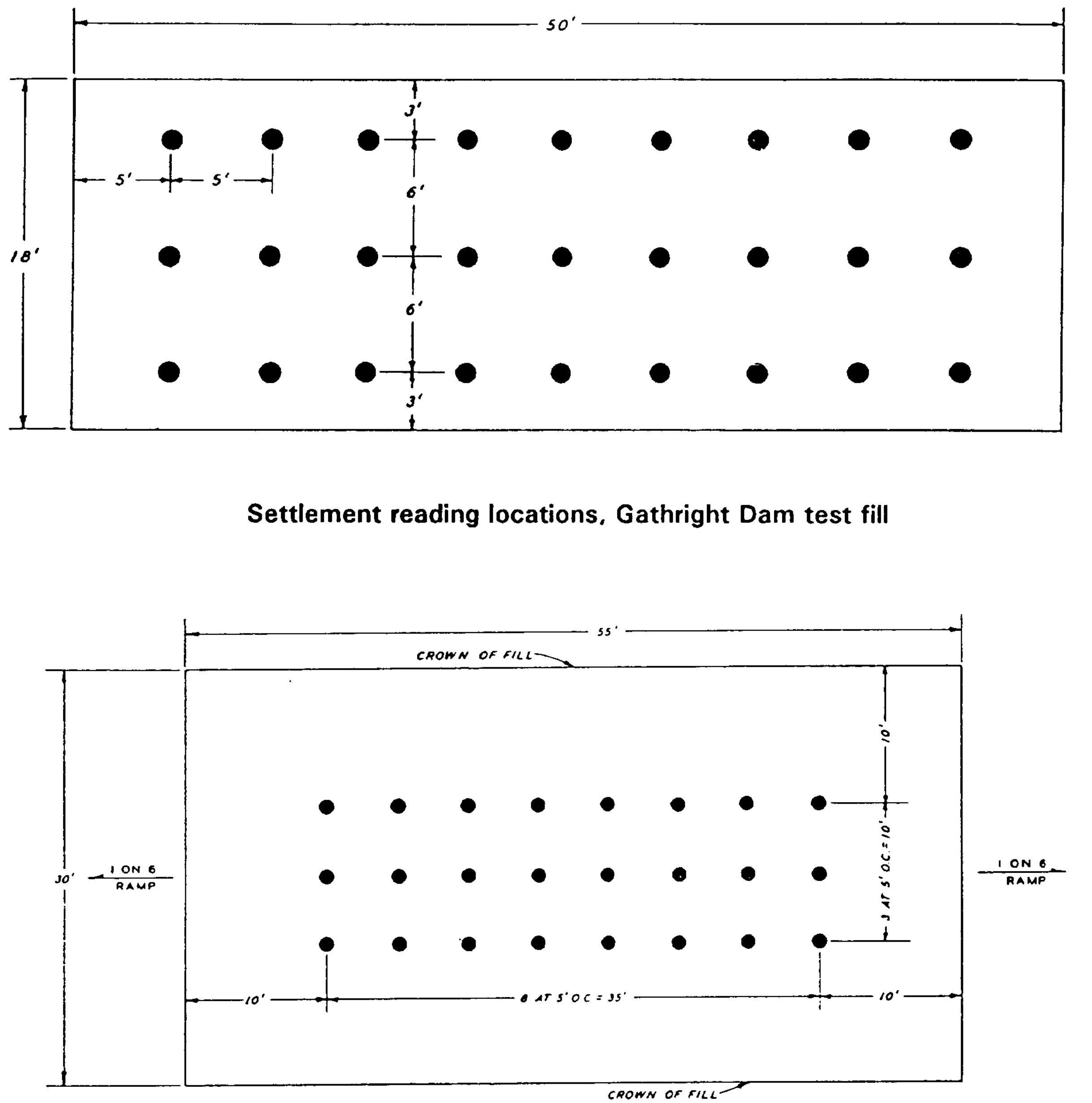
The densification of rockfill may be judged by: measuring the settlement resulting from compaction, performing in situ density tests, detailed observations within inspection trenches, and a combination of the preceding items. Because of the difficulty and expense of conducting enough tests to ensure representative results and because results of in situ density tests are sometimes questionable (especially for large rock), such tests should not be relied upon as the sole means of judging the effectiveness of the compaction process. Settlement determination by methods subsequently described should be used for this purpose in conjunction with visual observations in inspection trenches and with in situ density tests when available. In situ density tests are useful in that they provide quantitative values and allow comparison with the densities of other lift thicknesses or materials to be made but they are time-consuming and expensive to conduct.

a. Settlement. Settlement of the fill surface is measured by taking level readings at many points on the test section in a grid pattern. A 1.5- by 1.5-m (5- by 5-ft) square grid has often been used for this purpose. A 1.2- by 1.8-m (4- by 6-ft) or 1.5- by 2.1-m (5- by 7-ft) grid has also been used depending on the shape of the test fill area. Any gird pattern is acceptable as long as enough points are provided to obtain a good representative assessment of the overall settlement of the lift surface. There should be no less than 3 points on any one line of the grid and the edges of the grid should be no closer than 3 m (10 ft) from any outside edge of the test section to avoid settlement readings in an area where the rolling of the fill may have caused bulging. The areas to be avoided for settlement readings also include those next to the access ramps where lateral movement may also occur against the random, more compressible material often placed in those areas. Examples of settlement grid layouts are shown in Figure 11-1.

(1) Prior to establishing the grid points on the uncompacted lift surface, a leveling pass should be made by the vibratory roller with the vibratory unit off. This will provide a smoother surface upon which to establish and mark the grid points and to confirm the loose-lift thickness. This leveling pass with the vibratory roller can also be used when other types of rollers are to be assessed.

(2) There are several methods to establishing the grid. In most cases, wires or strings have been pulled from perimeter stakes set at the desired spacing. Another satisfactory method has utilized a light-weight template consisting of a metal frame strung with wire or twine. In any event, after the points are located, they should be well marked on the fill surface with contrasting paint to facilitate identification for subsequent level readings.

(3) Since the reading at a point must represent the area surrounding it (for points on 1.5-m (5-ft) centers, for instance, each point represents a 1.5- by 1.5-m (5- by 5-ft) area), it is important that the level rod be placed where it is indeed representative of this area and not influenced by local irregularities at the point. This is an expectable problem on a rockfill surface which can be ameliorated by use of the device shown in Figure 11-2. The simple device consists of a 0.3-m (1-ft) square metal plate with a raised button in its center upon which the level rod is seated for readings. A handle made from a steel rod is attached to the plate to help in firmly seating it and transporting it from point to point. The leveling instrument should be located carefully with regard to equipment trafficking so as to avoid its disturbance throughout placement and rolling operations. Bench marks should be established in secure places well away from the fill area. These are well known good survey



Settlement reading locations, Gathright Dam test fill

Grid for settlement readings, New Melones Dam test fill

Figure 11-1. Example settlement grid layouts

practices but reports on past test fills have attributed erratic settlement results to disturbance of the level instrument in more than a few cases.

(4) The settlement of a particular lift is obtained by averaging the settlements measured at all points in the grid usually expressing it as a percentage of loose-lift

Figure 11-2. Apparatus for taking level readings

thickness. If a number of passes of the roller is a variable being evaluated, readings of the grid points are usually made after every two passes or coverages of the roller. One pass of a pneumatic or vibratory roller is equivalent to one coverage. Those points that indicate heave rather than settlement may be eliminated from the averaging process if none of the surrounding points indicate heave or if it is noted (by careful observation and documentation by the rodman) that a local condition at the point is not representative of the surrounding area. If settlement plates have been installed beneath the fill on a compressible foundation, they should be read at the same frequency as the surface grid points, and any settlement they indicate averaged and subtracted from the lift surface settlement.

b. In situ density tests. The typical means of performing an in situ density test in rockfill is by excavating a test pit in the fill, salvaging and weighing the removed material, and determining the volume of the pit by the water-volume method. The volume of material excavated is necessarily large in order to minimize the effects of the larger particles on the results. The test pit may be either square or round in plan shape and a rigid template of wood or metal is anchored on the fill surface as a guide in excavating the hole. The template size and, therefore, the volume of the test pit varies on the basis of the maximum particle size in the compacted material. The most popular template seen in the rockfill literature has been round and metal. For that reason, in situ density tests in rockfill are often referred to as "ring densities." Since rockfill is segregated or stratified, the test pit should be excavated through the entire lift to obtain an average density. The use of lift marker material such as lime or plastic sheeting previously mentioned in paragraph 10-5 proves its worth in clearly indicating the lower lift boundary. Detailed guidance in conducting the test is now provided as ASTM Designation D 5030 - 89 (see ASTM 1994a). Weighing of the total sample removed form the density test typically requires high-capacity scales and may even involve weighing of individual larger particles. Rockfill materials are typically not placed under density and/or water-content specifications as are soils but are placed under method-type specifications with no water content control. The in situ density test is not used as a construction control test in project construction simply because of its time-consuming nature, the fact that rockfill density numbers have little specific meaning in design and the method-type specification coupled with good observation is relied upon for compaction quality. This is precisely why a test fill is so important, i.e., to aid in developing the proper method specification. The value of the test is primarily as a basis of comparison of compaction procedures in a test fill in establishing the method specification, i.e., lift thickness and number of passes by a specified roller, and to obtain as-built data during construction. Because of the heterogeneity of rockfill, in situ tests taken in the same lift at different locations can be expected to yield different results although more than one test per lift is rarely ever performed to reveal that expectable occurrence. In some cases, in situ tests have been employed in project construction, not only to obtain as-built data, but also when changes in materials or compaction results are suspected. U.S. Committee on Large Dams (1988) presents the experiences of several agencies, including the Corps of Engineers, with respect to in situ density testing.

c. Laboratory maximum density test. It has not been uncommon practice to compare the results of in situ density tests with the results of some version of vibrated laboratory maximum density. Both the Corps of Engineers (EM 1110-2-1906) and ASTM Designation D 4253-93 (ASTM 1994c) provide standard procedures for cohesionless materials with a maximum particle size of 7.6 cm (3 in.). EM 1110-2-1906 allows the scalping of up to 10 percent by weight of particles larger than 7.6 cm (3 in.) but ASTM requires that 100 percent of the material is smaller than 7.6 cm (3 in.). The literature such as U.S. Committee on Large Dams (1988) describe various non-standard large scale vibrated tests in large molds. The U.S. Army Engineer District, Los Angeles (1992) performed saturated tests in a 68.5-cm (27-in.) diameter mold on minus 15.2-cm (6-in.) scalped fractions of Seven Oaks Dam test fill gradations. The U.S. Army Engineer Waterways Experiment Station conducted an unpublished and limited test program on minus 15.2-cm (6-in.) rock employing standard tests in the 27.9-cm (11-in.) diameter mold on minus 7.6-cm (3-in.) scalped fractions and full-scale gradation tests in a 1.83-m (6-ft) diameter mold vibrated with an MTS® system actuator. That test series showed that the results of the standard test could be corrected for the oversize (plus 7.6-cm (3-in.) fraction) using the following equation of EM 1110-2-1911, Appendix B (Ziegler 1948):

$$\gamma_f = \frac{f\gamma_t\gamma_w G_m}{\gamma_w G_m - c\gamma_t} \tag{11-1}$$

or, solving for γ_t:

$$\gamma_t = \frac{\gamma_f\gamma_w G_m}{f\gamma_w G_m + c\gamma_f} \tag{11-2}$$

where, for this particular case:

γ_f = maximum dry density of the minus 7.6-cm (3-in.) fraction obtained from the standard EM 1110-2-1906, Appendix XII, vibratory table test, metric ton/m^3 or lb/ft^3

f = percentage by weight of the total sample represented by the finer fraction, i.e., the minus 7.6-cm (3-in.) fraction

γ_t = calculated maximum dry density of the total material, metric ton/m^3 or lb/ft^3

γ_w = unit weight of water, 1 metric ton/m^3 or 62.4 lb/ft^3

G_m = bulk specific gravity of the oversized fraction (plus 7.6-cm or plus 3-in.), see EM 1110-2-1906, Appendix IV

c = percentage by weight of the total material represented by the coarser fraction, i.e., the plus 7.6-cm (3-in.) fraction

11-3. Gradation Tests

Gradation tests are used to determine the amount of breakage the rock has suffered during placement and compaction. This is accomplished by running gradation tests on samples representative of the material as delivered to the fill ("before" gradations) and on samples taken from the compacted fill ("after" gradations). Differences in the two resulting curves indicate the degree of breakdown of the material. After-compaction samples are usually obtained from material excavated from the fill density test pit or from the side walls of inspection trenches or test pits. Again, it is important that the entire thickness of a lift be sampled. At the present time there is no standard procedure for obtaining the gradation of rockfill materials. A gradation sample must also be large in order to obtain a representative gradation curve with reasonable accuracy in the results. One approach to the concept of accuracy is to consider a test sample of such size that the addition or loss of the largest particle will not alter the "percent finer by weight" by more than an acceptable number of percentage points (i.e., shift the gradation curve coarser or finer). For instance, for a test sample to approach 1 percent accuracy, assume that the largest particle weighs 68 kg (150 lb). For this rock to correspond to less than 1 percent of the total sample weight, the total gradation sample size would have to be 6.8 metric tons (15,000 lb). In a similar manner, for 2 percent accuracy, the sample would have to weigh 3.4 metric tons (7,500 lb). From a practical point of view, a 1 percent accuracy is probably an extreme requirement and a 2 percent accuracy is a reasonable minimum criteria. The difficulty for rock materials lies in handling the heavy sample, obtaining its total weight, dividing it into fractions for gradation by different procedures, and then mathematically recombining the results on the fractions into a single gradation curve. A large, clean area (preferably a concrete slab) is needed to spread out the larger particle fraction. Determining the percentage by weight of total sample which the largest particles in various size ranges represent typically requires hand measurement in some manner of the size of larger particles and the summing of their weights for selected size ranges. Procedures for rocks larger than 12.7 cm (5 in.) now provided in ASTM Designation: D 5519-93 (ASTM 1994d) make be considered applicable for rock fill materials though specifically they are directed at riprap. For the total material fraction smaller than that treated by ASTM Designation: D 5519-93 (ASTM 1994d), gradation test procedures for aggregates such as Designation: C 136 (ASTM 1994b) and for soils such as Designation: D 422 (ASTM 1994c) or EM 1110-2-1906, Appendix V are available. The U.S. Committee on Large Dams (1988) describes the past large-scale gradation practices of several agencies which may also still be considered applicable. The procedures used in the construction of

Carter's, Cerrilllos, and Seven Oaks Dams were described previously in paragraph 6-5.

11-4. Percolation Tests

Rockfill in a dam shell is assumed to be a free-draining material in design. In cases where the material breaks down such that it exhibits poor to practically impervious drainage characteristics, the zoning of the embankment should ordinarily specifically provide for the use of such materials. It may be one of the objectives of the test fill to determine if such embankment design features must be incorporated to efficiently use available materials. For hard to medium rock, there is rarely a need to perform percolation (infiltration) tests to verify the free-draining characteristics. Assessments of the drainage characteristics of rockfill are very crude and are properly termed "percolation" or "infiltration" tests as opposed to "permeability" tests. Field methods applied in the test fill can only yield a very rough estimate of permeability because, among other factors, the material is unsaturated and the area of flow discharge is not known. Furthermore, the variability of the rockfill itself and that of permeability determinations (even under the best of laboratory conditions) would likely result in different values at different locations in the same lift of orders of magnitude. It has become customary to describe soil-rock materials with permeabilities less than 0.3 m/year (1 ft/year) as impervious, those with permeabilities between 0.3 and 30 m/year (1 and 100 ft/year) as semipervious, and those with permeabilities greater than 30 m/year (100 ft/year) as pervious (U.S. Department of the Interior, Bureau of Reclamation 1985). These ranges were derived from Terzaghi and Peck (1948) who described them as representing good drainage characteristics if permeability is greater than 10^{-4} cm/sec (103 ft/year), poor drainage characteristics if permeability is between 1×10^{-6} cm/sec and 1×10^{-4} cm/sec (1 and 103 ft/year), and practically impervious if permeability is less than 1×10^{-6} cm/sec (1 ft/year). The following paragraphs describe several versions of a percolation test given in the order of decreasing applicability for most test fill situations.

a. Open pit method. This method has been by far the most commonly used. Percolation tests are usually performed in either the in situ density test hole after removal of the plastic liner employed to obtain the water-volume or in a separate pit at least 1-m (3-ft) square and at least one lift thickness deep. If the pit will not retain a sufficient volume of water to measure the rate of fall of the water surface, the compacted material is obviously free-draining. If the pit can be filled such that a rate of fall of the water surface can be measured, the percolation rate (the distance the ponded water surface falls in the pit over a measured time) in centimeters/second, meters/day or ft/day may be taken directly as a very rough indication of the permeability. This simple approach will overestimate the permeability and further guidance concerning its use given in paragraph 11-4*c* below should be considered. However, if this crude method indicates that the drainage characteristics are less than free-draining, it is a reliable assumption that they are. Justo (1991) provides a more theoretical method to estimate a permeability from the rate at which the water surface in a test pit falls, i.e., a falling-head test. The permeability is calculated with reference to Figure 11-3 and using the following equation. An example calculation is given below in paragraph 11-4*c*.

$$k = -\frac{\Delta h}{\Delta t\ (1 - n\ \Delta S)} - \frac{h_0\ n\ \Delta S}{\Delta t\ (1 - n\ \Delta S)^2}\, ln\frac{h\ n\ \Delta S - \Delta h}{h_0\ n\ \Delta S} \tag{11-3}$$

where, for any consistent set of units:

k = coefficient of permeability

h_0 = the initial depth of water in the pit at time $t = 0$ sec

h = depth of water in the pit after a time Δt sec

Δh = $h_0 - h$ = change in depth of water in the pit during time interval Δt sec. **Δh takes a negative sign in the above equation because in its derivation, as the head decreases during the test, the volume of water in the material from the bottom of the pit to the wetting front is increasing.**

n = the porosity of the fill beneath the pit, dimensionless

ΔS = change in degree of saturation (expressed as a decimal value) of the material below the pit from its initial value before filling of the pit to its wetted value during the test.

This approach assumes that the pit is filled instantaneously which is not the practical case and the change in degree of saturation must be estimated. Furthermore, flow is assumed to take place only in the vertical

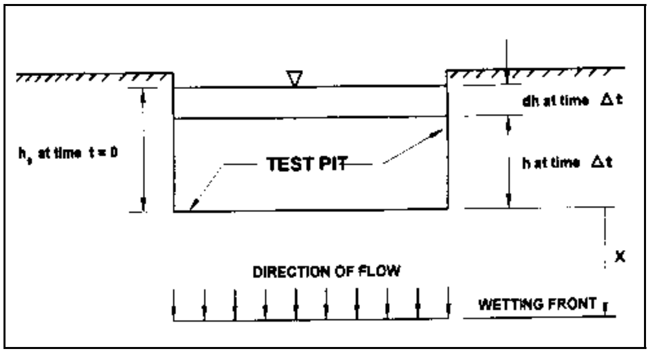


Figure 11-3. Open-pit percolation test after Justo (1991)

direction such that at any distance x below the pit (see Figure 11-3), the area of discharge is not assumed to have increased. This assumption represents a sever simplification of the flow pattern since flow will occur through the side walls of the pit below the water level and will also spread out laterally beneath the pit with distance x in downward movement of the wetting front. Since increase in discharge area with distance below the pit is not considered, the calculated value of permeability will be larger than the actual value to some unknown degree assuming that all other parameters entered into the equation are correct. The initial degree of saturation of the fill and its porosity can be calculated according to EM 1110-2-1906, Appendix II, from a knowledge of the initial water content of the fill, the specific gravity of the material, and the compacted density of the fill. This information can be obtained as part of the conduct of a fill-density test. If the fill contains more than about 10 percent by weight of particles passing the U.S. Standard No. 4 sieve, the specific gravity of the material should be calculated using the equation given in EM 1110-2-1906, Appendix IV, paragraph 3*e*, for materials consisting of both plus and minus No. 4 sieve fractions. The wetted degree of saturation of the fill beneath the test pit at the time Δt during conduct of the percolation test must be estimated. The wetted degree of saturation may vary between about 75 and 95 percent. A reasonable value to assume for the computation of permeability is 85 to 90 percent.

b. Standpipe methods. The standpipe methods to be described below consist of using a cased hole (i.e., an implanted metal or plastic pipe in the case of the rock test fill) to perform either a constant-head or falling-head permeability test. The falling-head standpipe test is probably the most technically sound method among the four methods presented because it does consider the increase in area of flow (to be discussed below) after a fashion proven in model studies. The standpipe tests are applicable in materials exhibiting many times the permeability identified previously as indicating good drainage characteristics. However, they should not be used unless the fill contains appreciable fines and the considerable time and costs are deemed to be justified. The fill should probably exhibit a minus No. 4 sieve fraction exceeding 30 percent by weight or a minus No. 200 sieve fraction of 10 percent or more by weight in order that it not be so permeable as to outstrip the practicality of a standpipe test. If the material is gap-graded, the question of its permeability outstripping the practicality of the standpipe test is complex but if the finer fraction does not fill the voids between the larger particles, the permeability is likely to be very high. There are no firm guidelines concerning the diameter of the pipe but ideally it should not be less than twice the diameter of the maximum particle size of the material after compaction. If this criterion dictates a very large (in practical terms) pipe diameter, say, exceeding about 61 cm (2 ft), a ratio of diameter to maximum particle size of less than two may be used but with care taken to avoid a single particle immediately below the tip of the pipe which would constrict flow. The objective is to test the mass of the compacted rock to the maximum practical extent. The pipe should be inserted into the fill to a sufficient depth such that it is stable in position. In general, the pipe must be placed in an excavation kept to minimal working dimensions and then backfilled about its exterior with the excavated material with some attempt to maintain its approximate gradation as it was in situ. Material above the level of the tip of the pipe has some impact on results because the flow of water has been shown by model tests to develop a wetting front in the unsaturated material which is approximately spherical about the tip of the pipe.

(1) Schmid (1967) provides a method for estimating permeability from the results of a falling-head test performed above the ground water table (in unsaturated material) in a standpipe. Because the volume of the standpipe is small compared to the ability of the material to consume the flow, the depth of pipe embedment need only be that sufficient to stabilize it. Figure 11-4 shows the configuration of the test and the spherical wetting front (documented by model tests) which develops as the water flows from the pipe. The equation Schmid (1967) derived is given below. An example calculation is given in paragraph 11-4*c*.

$$k = \frac{r_0}{4} \frac{\ln \dfrac{h_1}{h_2}}{t_2\left[\dfrac{3\,(h_1 - h_2)}{4\,n\,\Delta S\,r_0} + 1\right]^{\frac{1}{3}} - t_1} \tag{11-4}$$

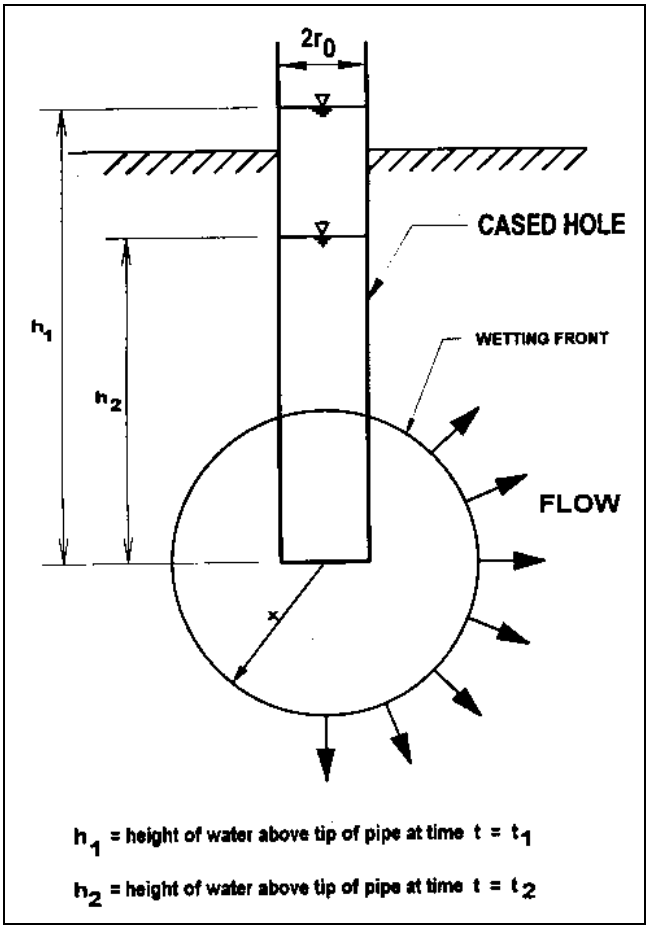


Figure 11-4. Falling-head standpipe percolation test after Schmid (1967)

where for any consistent set of units:

k = coefficient of permeability

r_0 = inside radius of the standpipe

h_1 = height of the water above the tip of the standpipe at time $t = t_1$

h_2 = height of the water above the tip of the standpipe at time $t = t_2$

ΔS = change in degree of saturation of the rockfill from its initial value to its wetted value expressed as a decimal value.

n = the porosity of the rockfill

The wetted degree of saturation can again be assumed to be 85 to 90 percent as previously stated in paragraph 11-4*a*.

(2) The U.S. Bureau of Reclamation in its *Earth Manual* (USBR 1985) provides an open-end field permeability measurement method as Designation E-18. The method consists of measuring the flow rate of water required to maintain a constant head in the pipe under a constant rate of flow. Gravity flow in an open standpipe or pressurized (pumped) flow in a sealed standpipe may be used. For this test, the pipe must be inserted into the fill to a depth such that the head of water applied as measured from the tip of the pipe is less than the depth of embedment. Otherwise, the flowing water may rise about the pipe to exit upon the surface of the fill which will invalidate the already approximate test method. This constant-head test is complicated for use in rock test fills by the need to measure and maintain the flow rate, and by the necessity to maintain either a relatively constant water level in the pipe (gravity flow) or a relatively constant pressure (pressurized flow). In tests using gravity flow, a constant water level in the standpipe is rarely maintained in unsaturated materials and a surging of the level within less than 15 cm (6 in.) at a constant rate of flow for about 5 minutes is considered satisfactory. In the pressurized test, this acceptable head variation corresponds to a pressure variation of only about 21 kPa (3 psi). The equation used to calculate permeability was derived by the USBR from electrical analogy studies and is given below (see Figure 11-5). An example calculation is given in paragraph 11-4*c*.

$$k = \frac{Q_d}{5.5\ r_0\ H_w} \tag{11-5}$$

where, for any consistent set of units:

k = coefficient of permeability

Q_d = flow rate

H_w = applied head

r_0 = inside radius of the standpipe

c. A comparison of the methods. Justo (1991) compares results of open-pit and standpipe percolation

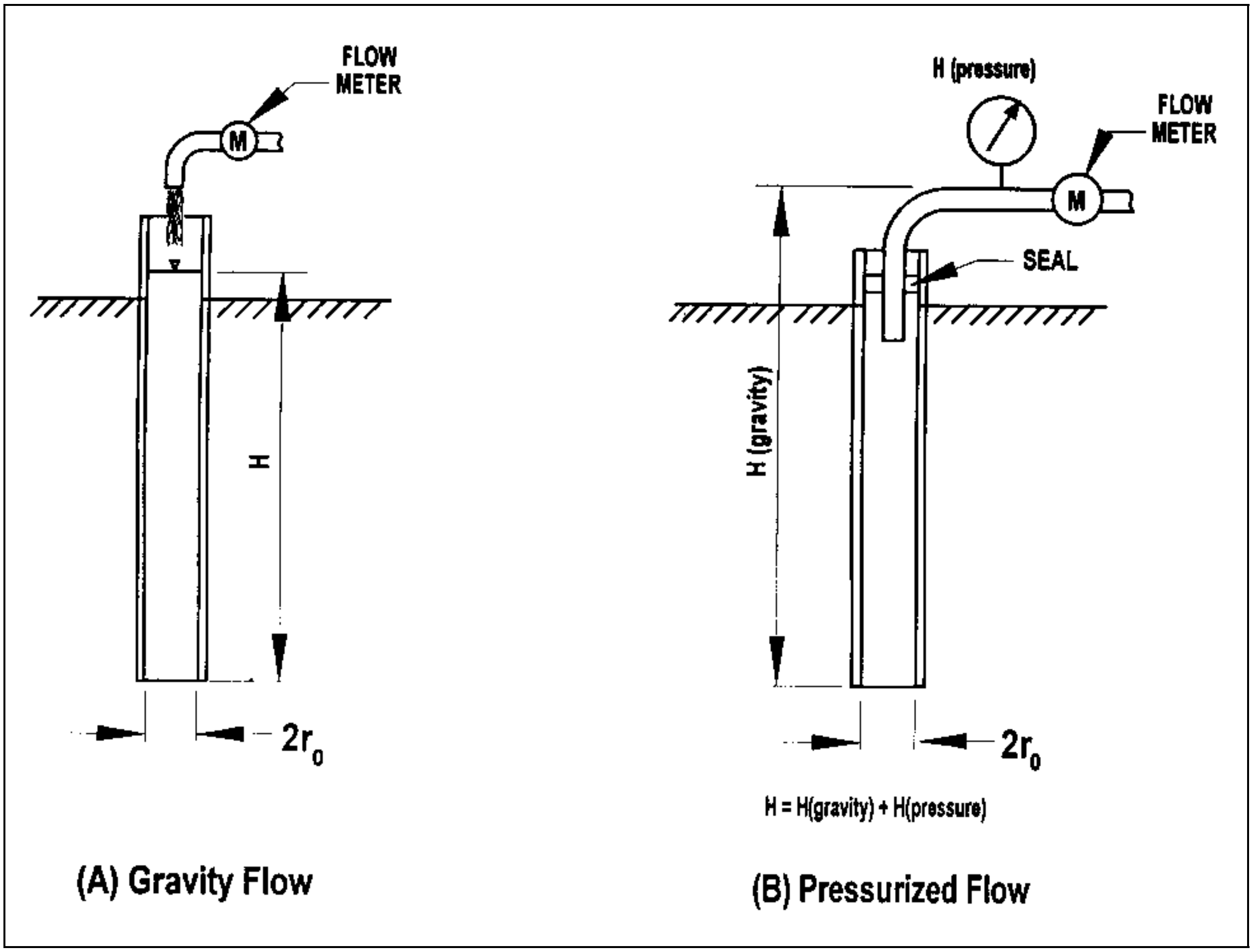


Figure 11-5. USBR gravity flow and pressurized flow standpipe percolation tests

tests performed in the test fill for Martin Gonzalo Dam, Cordoba, Spain. The open-pit method was conducted in a pit excavated to dimensions of 1m square by 0.25 m deep (3.3-ft square by 9.8 in. deep). Both the USBR constant-head, constant-flow test and the falling-head test were performed in standpipes. The permeabilities according to the equations (11-3) through (11-5) presented above in paragraphs 11-4*a* and 11-4*b* are calculated below.

(1) The open pit was filled to an initial depth of 25 cm (9.8 in.), i.e., to the surface of the test fill. Parameters pertinent for entry into Equation (11-3) of paragraph 11-4*a* were as follows:

dry density of the fill = 2.14 Mg/m^3

porosity of the fill = n = 0.20

initial degree of saturation S_i = 10 percent = 0.10

wetted degree of saturation S_w = 90 percent = 0.90

$$\Delta S = (S_w - S_i) = (0.90 - 0.10) = 0.80$$

$$1 - n\Delta S = 1.0 - (0.20)(0.80) = 0.84$$

$$h_0 = 25 \text{ cm}$$

Δh during conduct of the test was 11 cm (-4.33 in.) over a time period Δt = 180 sec. It is important to apply a minus sign to Δh in the equation (11-3). Because Δh observed was 11 cm (4.33 in.), h at time Δt = 180 sec = $h_0 - \Delta h$ = 25 cm - 11 cm = 14 cm (5.51 in.)

Substituting into Equation (11-3):

$$k = -\frac{-11}{180 \times 0.84} - \frac{25 \times 0.16}{180 \times (0.84)^2} \, ln \, \frac{(14 \times 0.16) - (-11)}{25 \times 0.16} \tag{11-6}$$

or:

$$k = 0.0728 - 0.0377 = 0.0351$$
$$= 3.51 \times 10^{-2} \text{ cm/sec} = 1.1 \times 10^4 \text{ m/yr}$$

The material is free-draining according to the criteria discussed in the first part of this section because the measured permeability is a great deal in excess of 30 m/year (103 ft/yr).

(2) The parameters from the Martin Gonzalo Dam tests pertinent for substitution into Schmid's Equation (11-4) of paragraph 11-4*b*(1) were as follows:

As before

$$n\Delta S = 0.16$$

radius of the standpipe $r_0 = 10.25$ cm

$$h_1 = 40 \text{ cm}$$

$$h_2 = 20 \text{ cm}$$

$$t_1 = 0$$

$$t_2 = 192 \text{ sec}$$

substituting into Equation (11-4):

$$k = \frac{10.25}{4} \frac{\ln 2}{192 \left[\frac{3 \times 20}{4 \times 0.16 \times 20.5} \quad 1 \right]^{1/3}} \tag{11-7}$$

$$k = 5.2 \times 10^{-3} \text{ cm/sec} = 1.64 \times 10^3 \text{ m/yr}$$

(3) With respect to the USBR standpipe method, the constant flow measured for a constant head of 30 cm was 34.38 cm^3/sec in the same standpipe with r_0 = 10.25 cm. Substituting into equation (11-5) yields:

$$[k = \frac{34.38}{5.5 \times 10.25 \times 30} = 2.03 \times 10^{-2} \text{ cm/sec}$$
$$= 6.40 \times 10^3 \text{ m/yr}$$

The values of permeability yielded by the three tests given above vary by about one order of magnitude (one power of 10) which is an excellent result considering the differences in the tests and the partially saturated state of the fill. The permeability of a partially saturated soil is very sensitive to the pore space filled with water or the so-called "effective porosity." Even permeability tests performed on replicate specimens (specimens carefully prepared to be as identical as practicable) of a saturated soil in a laboratory environment using different methods of test and calculation of coefficient of permeability may yield results of greater variation than seen in the above examples. It is instructive to note from the open-pit test above that if the permeability is estimated by simply the fall of the water Δh = 11 cm divided by the time lapse Δt = 180 sec, i.e.:

$$k = \frac{11}{180} = 6.11 \times 10^{-2} \text{ cm/sec} \tag{11-8}$$
$$= 1.93 \times 10^4 \text{ m/yr}$$

the resulting number is on the order of 2 times the value yielded by the Justo open-pit method to 10 times the value obtained from the Schmid falling-head standpipe method. Since the Justo method overestimates the permeability for reasons previously stated, it is recommended that the permeability estimated directly by fall of the water Δh divided by the time lapse Δt be reduced by a factor of 10 in judging the actual drainage characteristics of the fill.

11-5. Other Tests

It is beyond the scope of this manual to provide a treatise on the subject of rock testing. Most current laboratory tests do not have direct correlative relationships to rockfill with respect to either rock quality specifically as a fill material or its engineering properties or behavior. For high-quality, hard to medium rockfill materials, laboratory shear tests, compressibility tests, and associated stability analyses are typically not performed for the embankment itself and embankment slopes are adopted at 1 vertical on 3 horizontal since the shear strength of sound rockfill is well established to be at least ϕ = 45 degrees. The designer must make a decision regarding rockfill which breaks down significantly as to whether it still retains rockfill strength or must be treated as a soil containing large particles, in which case embankment stability analyses may be required. Of course, in the event that there exists the possibility that embankment failure may involve the foundation, stability analyses are conducted for those cases regardless of the quality of the rockfill. The comments provided below are very general and divided into the categories of laboratory tests and field tests.

a. Laboratory tests. There are a number of index-type tests which may be performed on rock to obtain a judgment of its mechanical or environmental durability. In addition to the tests previously mentioned in Part 1 of this manual (some are ASTM Standards), there is the Los Angeles Abrasion test for large aggregate described in ASTM Designation C 535-89 (ASTM 1994b). That test is restricted to minus 7.6-cm (3-in.) particle sizes and has limited usefulness in indicating the likely breakdown of material under placement and compaction operations. Hammer and Torrey (1973) attempted to analyze available data from test fills to correlate Los Angeles Abrasion data to degradation during handling and compaction. Their attempts were not successful at that time and no subsequent correlations are known to exist based on the improved procedures of ASTM C 535-89. At least it is reasonable to assume that rock which suffers serious degradation in the test will suffer significant breakdown in rockfill operations. EM 1110-2-2302 lists several tests and criterion for suitability of stone for general construction use (see Table 11-1). Those criterion are very general and should be considered to indicate the higher quality rockfill materials which may be relatively obvious without test results in many cases and, therefore, do not help much in prediction of rockfill qualities of softer materials. Other references pertinent to rock and rockfill quality testing are Lutton, Houston, and Warriner (1981), U.S. Army Engineer Waterways Experiment Station (1993), and NATO ASI Series (1991). Donaghe and Torrey (1985) and Torrey and Donaghe (1991) treat the shear strength and compaction characteristics of earth-rock mixtures (soil materials). Lutton (1977) and Strohm, Bragg, and Ziegler (1978) address shale materials which may appear as rockfill upon excavation and placement but which degrade with wetting into a soil.

b. Other field tests. In situ tests (other than in situ density and percolation tests) may be performed in the test fill to assess the strength and compressibility of the compacted material although such tests have very rarely been used in this country. As described by Justo (1991), compressibility has been assessed by large plate load tests and shear strength has been measured by plate load tests taken to failure and in situ passive failure shear tests (jacking a vertical plate against a vertical rockfill face).

11-6. Visual Observation

Because of the nature of a test fill, visual observation of the various construction procedures and material behavior are very important as a source of qualitative supplemental information. Some items meriting close observance are: preparation of the leveling pad before fill construction, installation procedures for any instrumentation such as settlement plates, character of the rock delivered to the fill such as consistency in gradation and condition, breakage of the rock during spreading relative to the degree of working by the crawler tractor, breakage of the rock during compaction, effects of added water (if any), smoothness of the surface after each interval of rolling, appearance of the fill during and after rainfall, and any variation in established behavior of any phase of the construction operation. All visual observations should be well documented with photographic evidence and a written record.

11-7. Inspection Trenches or Pits

It is highly desirable to expose a cross section of each test section or lane in order that general in situ characteristics of the compacted fill might be observed. This is achieved by the excavation of pits or inspection trenches. The inspection pit is excavated from the top down through a lift immediately after rolling or through all or part of the lifts after the entire test section or lane is completed. An in situ density test excavation can also serve as an inspection pit. If separate inspection pits are employed through several lifts, they should be large enough in plan area to permit the safe presence of personnel to inspect the side walls and even take samples. An inspection trench is a cut made through the entire depth and usually across the entire width of the completed test section or test fill. Excavation is normally with a front-end loader or dozer. The inspection trench is most often used, since the only advantage of an inspection pit is that it can be dug at any stage during the rolling operation, but this is not often justified. Except as a source of after-compaction gradation samples, an inspection trench is primarily for qualitative examination such as amount of rock-to-rock contact of the compacted material. Figure 11-6 shows such an inspection trench cut through the test fill for New Melones Dam. That trench was excavated approximately along the line of cross section A-A of Figure 9-5. Note

Table 11-1
Criteria for Evaluating Stone (after EM 1110-2-2302)

Test	Approximate Criterion for Suitability*
Petrography	Fresh, interlocking crystalline, with few vugs, no clay minerals, and no soluble minerals
Unit Weight	Dry unit weight 160 lb/cu ft or greater
Absorption	Less than 1 percent
Sulfate Soundness	Less than 5 percent loss
Glycol Soundness	No deterioration except minor crumbs from surface
Abrasion	Less than 20 percent loss for 500 revolutions
Freezing-Thawing	Less than 10 percent loss for 12 cycles
Wetting-Drying	No major progressive cracking
Field Visual	Distinctions based on color, massiveness, and other visual characteristics
Field Index	Distinctions based on scratch, ring, and other physical characteristics
Drop Test	No breakage or cracking
Set Aside	No breakage or cracking after one season cycle

* Marginal test results usually indicate the need for supplemental testing for definitive evaluation

from Figure 11-6 that the trench is of such a width compared with the maximum height of fill as to ensure the safety of personnel moving about within it to observe the side walls or take samples. Regardless of the size of inspection pits or trenches, personnel entering them should be screened for the proper and usually required construction safety equipment such as steel-toed footwear, hard hats, and safety eye wear in the event that a heavy particle or shower of smaller material unexpectedly falls. This is particularly important since the side walls are typically loose and covered with some fall-out material requiring hand work (accomplished as excavation proceeds) to obtain a view of representative in situ material. If after-compaction gradation samples are to be taken from the side walls, the non-representative fall-out material must be excluded from the sample. The use of lift-surface markers such as lime or plastic sheeting (as previously discussed) are important to permit the pit or trench inspectors to clearly distinguish one lift from another. Items of interest when inspecting a pit or trench include: depth of fines on the lift surfaces, distribution of fines within the lift, the overall appearance of each lift such as the segregation pattern, occurrence of voids, and the general "tightness" of the fill, including the stability of the side walls. Thorough documentation of the inspection trench is very important including photographs and a written record. It should be considered imperative that design personnel take part in the inspections.

Figure 11-6. Inspection trench through the New Melones Dam test fill